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Design of Steel Structures
ECIV4318
Second semester 2014-2015
Mid- Term Exam 18 /4 /2015
Allowable Time: $1^{hr}\,15^{min}$

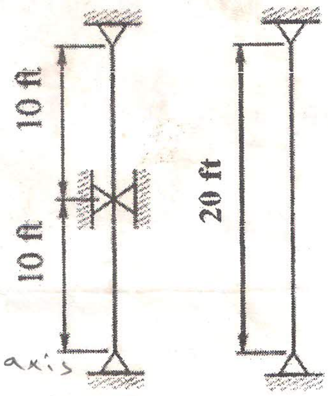
Student Name:-………………

Open AISC Manual Only

Question (1) (10) **10 Marks**

A design calculation is brought to you for a **20-foot** column, pinned at top and bottom with lateral bracing in the weak direction at mid-height. The factored load, P_u, is correctly calculated to be **750 kips**. The column design calculation uses a **W10×68** section of A992 steel (fy = 50ksi). After you have done some calculations you found a mistake that makes the **W10×68 is not adequate**:

(a) What was that mistake? Correct the mistake and Find the lightest **W10** section that is adequate.



$P_u = 750$ $\quad \frac{K_x L_x}{r_x} = \frac{1 \times 20 \times 12}{4.44} = 54.05$

$r_x = 4.44$

$r_y = 2.59$ $\quad \frac{K_y L_y}{r_y} = \frac{10 \times 12 \times 1}{2.59} = 46.33$

you think it is y axis is control but it is x axis

so you go and select lightest W10.

$\frac{K_x L_x}{r_x/r_y} = \frac{20}{1.71} = 11.69$ / use W10×77

$\frac{K_y L_y}{r_y} = \frac{10 \times 12}{2.6} = 46.15$ / $\frac{K_x L_x}{r_x} = \frac{20 \times 12}{4.498} = 53.35$ so $K_x L_x$ control ok

(b) Calculate the design compressive strength of the column designed in section (a) if **elastic** buckling behavior is controlling? E =29000 ksi

$\frac{K_x L_x}{r_x} = 53.35$

$F_e = \frac{3.14^2 \times 29000}{53.35^2} = 100.45$ kips

$\frac{f_y}{F_e} = \frac{50}{100.54} = 0.49 < 2.25$ inelastic

if elastic will control I calculate it = 1799 kips

$F_{cr} = 0.658^{50/100.45} \times 50 = 40.59$

$\phi P_n = 0.9 \times 40.59 \times 22.7 = 829.25$

ok

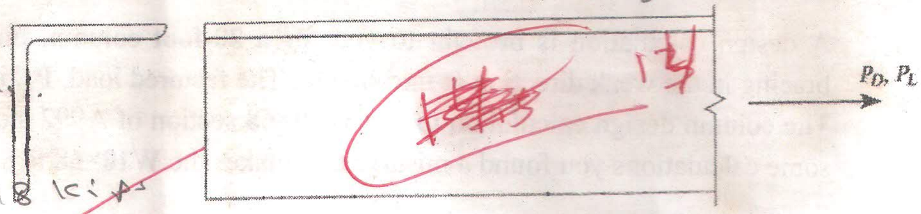
Question (2)

25 Marks

The **single-angle** tension member A36 steel (Fy=36 ksi, fu=58 ksi) shown below is connected using **5/8 inch diameter Group (A) bolts**. The dead load is $\mathbf{P_D = 35}$ **kip**, $\mathbf{P_L = 35}$ **kip**. The thickness of gusset plate is **3/8 inch**

a) Design an adequate **bearing** type connection and select the most economical **equal Leg angle** among L4, L5. The **connection only from one leg Don't use** staggering in Bolts, Ignore block shear capacity of the connection. Use **suitable line(s) of bolts**

b) Regardless of the angle that you selected in part (a), assume you are using an angle with unequal legs. Would you prefer to connect the bolts using the longer or shorter leg? Explain your answer.



$F_{nv} = 54$

assume shear in threads.

$P_u = 1.2 \times 35 + 1.6 \times 35 = 98$ kips

$\phi R_n = 0.75 \times 54 \times \frac{5^2}{8^2} \times \frac{3.14}{4} = 12.41$ kips

\# bolts $= \frac{98}{12.41} = 7.89 \simeq 8$ bolts

$A_g = \frac{98}{0.9 \times 36} = 3.02$

$A_e = \frac{98}{0.75 \times 54} = 2.41$

we use L4x4x $\frac{7}{16}$ = weight 11.3

$A_g = 3.3$

we us L5x5x $\frac{5}{16}$ weight 10.3

$A_g = 3.07$

so I use L5x5x $\frac{5}{16}$

b) I will use big one

1- becaus estability

2- I bi

16

L4/L6

1 1/4

-1 Upper

b Ei

2

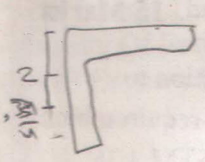
-1 rmi

Spacing -2

Ae -2

2

From manual



$U = 1 - \frac{\bar{x}}{L} = 1 - \frac{0.35}{3 \times \frac{5}{8} \times 3 + 1.25} =$

$= 0.95$

bearing behind the bolt will occur in angle (interior)

L_c between bolt $= 3 d_b$

$\phi R_n = 0.75 \times 1.2 \times (3 \times \frac{5}{8} - (\frac{1}{16} + \frac{5}{8}) \times \frac{5}{16} \times 58 = 19.37$

upper limite

$\phi R_n = 0.75 \times 2.4 \times \frac{5}{8} \times \frac{5}{16} \times 58 = 20.39$

bearing behind the bolt in (edge)

-1.0

$\phi R_n = 19.29 / 20.39$

$\phi R_n = 0.75 \times 1.2 \times (1.25 - (\frac{5}{8} + \frac{1}{16})/2) \times \frac{5}{16} \times 58 = 14.7$

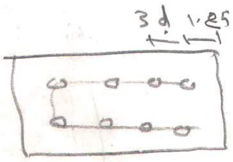
still shear control

check block shear

$A_{nt} = (1.75 - \frac{5}{8}) \frac{5}{16} = 0.35 \text{ in}^2$

$A_{gv} = (3 \times \frac{5}{8} \times 3 + 1.25) \times 2 \times \frac{5}{16} = 4.29 \text{ in}^2$

Ignore Block Shear



$A_{nv} = 4.29 - 7 \times \frac{5}{8} \times \frac{5}{16} = 2.92 \text{ in}^2$

$\phi R_n = (0.6 \times 58 \times 2.92 + 1 \times 58 \times 0.35) \times 0.75 = 91.43$

upper limit

$\phi R_n = (0.6 \times 36 \times 4.29 + 1 \times 58 \times 0.35) \times 0.75 = 84.72$

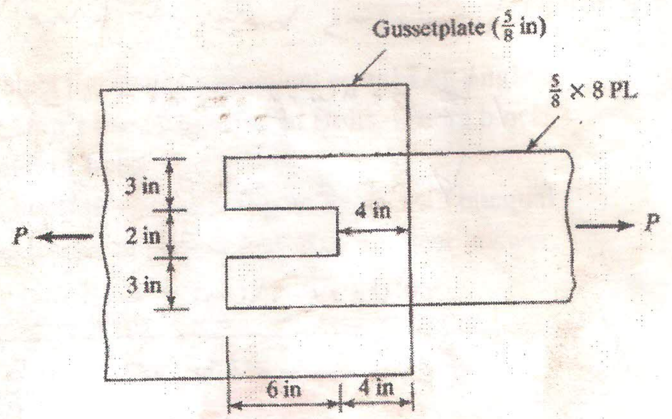
you need L_c bigger than $3 d_b$.

check U.

13.6

Question (3) **15 Marks**

The **5/8 × 8 – in PL** shown is to be connected to a gusset plate. Design the welded connection to develop the **full capacity of the tension plate**. Use the **minimum weld size** of the AISC requirements (Don't exceed w_{min}). E70 electrodes are to be used. All of the Structural Steel is to be ASTM A36 (F_y=36 ksi, f_u=58 ksi), **Ignore Block Shear.**



yeild member tension

$\phi P_n = 0.9 \times 6 \times \frac{5}{8} \times 36 = 121.5$ kips

−1

fracture member

$\phi P_n = 0.75 \times 6 \times \frac{5}{8} \times 58 = 163.125$

so yeild strength constrch

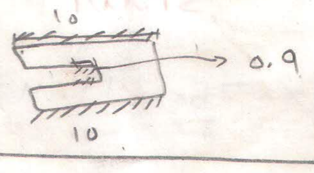
min w = $\frac{1}{4}$

$\phi R_n = 0.75 \times 0.707 \times \frac{1}{4} \times 0.6 \times 70$

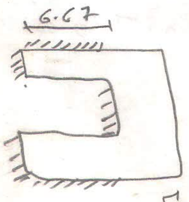
$= 5.56$ / 1" L / $\frac{1}{4}$" w

length of weld = $\frac{121.5}{5.56}$ = 21.8 inch

phase #1



phase #2



$\phi R_{nt} = 5.56 \times 7 = 38.92$

$\phi R_n = 0.85\ \phi R_{nL} + 1.5\ \phi R_{nt}$

$121.5 = 0.85 \times \phi R_{nL} + 1.5 \times 38.92$

$\phi R_{nL} = 74.25$

length of weld long = $\frac{74.25}{5.56}$ = 13.35

phase 2 = 13.35 + 7 = 20.35

so I choose phase 2